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Orphan Breakwaters – what protection is given when they collapse?

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Abstract

Many old coastal harbours in the UK are protected by blockwork breakwaters, but the original streams of income for maintaining or refurbishing these structures have largely diminished. Their breakwaters may, however, still protect harbour-side properties against wave overtopping, and thus flooding. This paper presents results from an exploratory study to identify how blockwork breakwaters common in many smaller UK coastal harbours may collapse due to storm action, and how much wave protection is given by collapsed breakwaters.

1. INTRODUCTION

1.1. Background

There is a long tradition in the UK of coastal towns or country estates constructing their own small harbours for trade and/or to shelter fishing boats. Such harbours were particularly needed on 'rocky' coastlines where the local topography / geology hindered construction of roads or railways. In the expansion of harbour construction around 1770-1880, many such harbours were protected against wave action by a breakwater (or multiple breakwaters) to reduce wave agitation, thus protecting quays, cargo handling facilities, and storage areas. These structures were commonly formed by rubble mounds built to low-water, surmounted by vertical or battered walls of dressed stone blocks (later concrete) with random rubble core between. The core (or hearting) of these structures is poorly documented, probably quarry-run interspersed with broken blocks and chiselled scraps of the wall-stones.



Figure 1: Wave attack on the Heugh breakwater at Hartlepool

[Source: nearthesea.co.uk]

For many such harbours, maritime incomes have since abated, especially with the movement of trade to rail or road, and with the diminution of fishing fleets. Many small harbours have, therefore, been left with little or no income against which to defray costs of maintaining / repairing their breakwater(s). If the

only beneficiaries of wave protection from these 'orphan' breakwaters were the original commercial operations, this lack of resources might have relatively little consequence. But, in the decades since their original construction, the areas protected from wave action have been increasingly adopted for commercial and/or residential purposes. A potential problem then arises from the absence of funding for maintaining the breakwater, to be set against the risk of structural failure or damage. If the 'orphan' breakwater were to collapse, the protection afforded to areas sheltered from direct wave action will reduce. This was discussed for Hartlepool (Figure 1) by Hampshire *et al.* (2013). Even after collapse, some wave reduction will still be afforded by the relict structure. In assessing the overall degree of protection, the level of wave transmission over the collapsed structure will be key.

Example breakwater cross-sections from St Catherine's and Whitehaven are shown in Figure 2, Blyth and Alderney in Figure 3, Hartlepool and Peterhead in Figure 4.

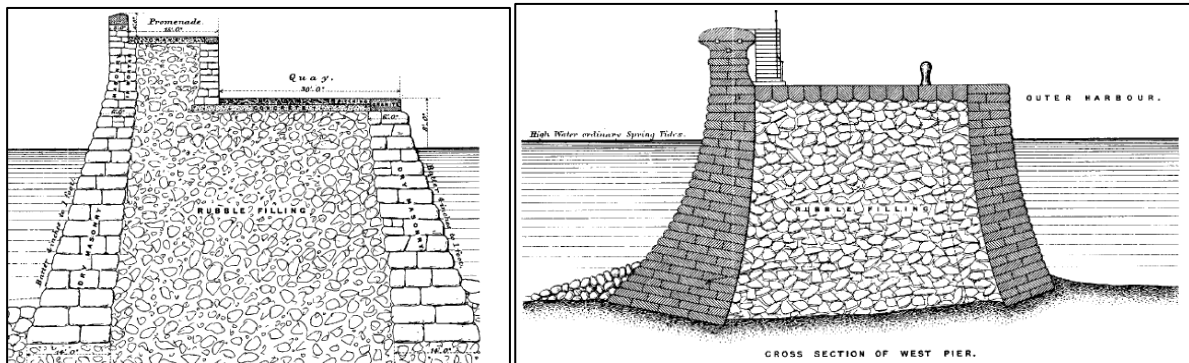


Figure 2: St Catherine's (left) and Whitehaven (right) breakwaters.

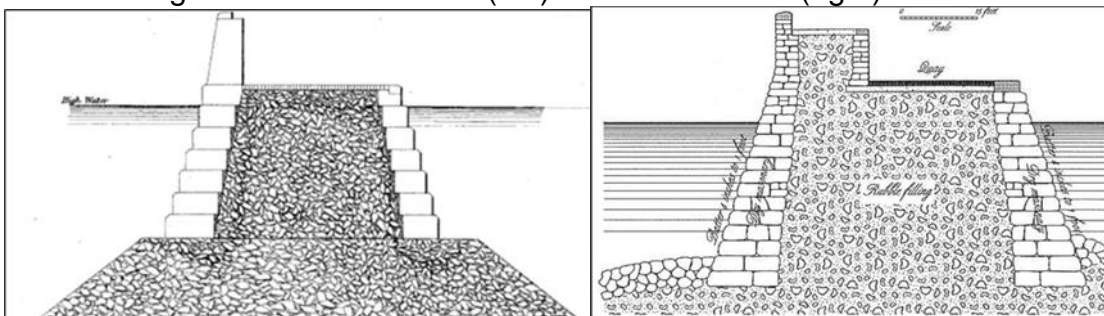


Figure 3: Blyth breakwater (left); Alderney breakwater (right)

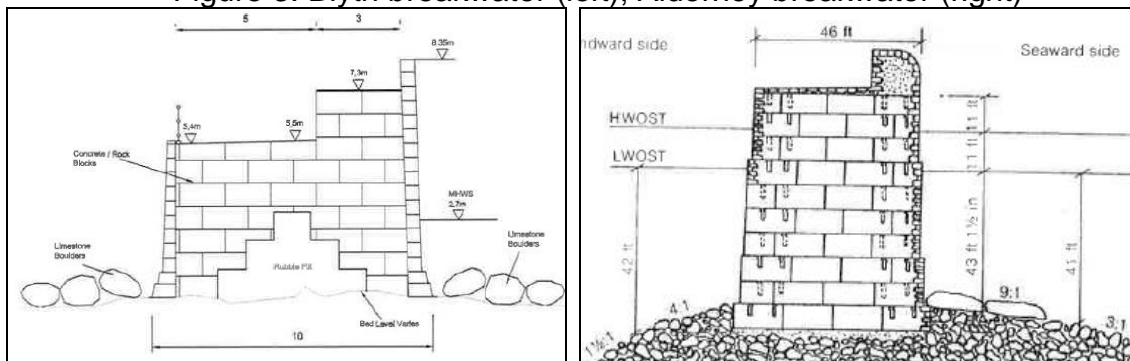


Figure 4: Hartlepool breakwater (left); Peterhead (right)

[Sources: Bray & Tatham (1992); Williams (1878); Scott (1858); Vernon-Harcourt (1873); Hampshire *et al* (2013); Buchan (1986)]

1.2. Origins of this project

Many breakwaters for minor harbours remain the primary sea defences for their harbours. But if their income no longer supports essential maintenance / repair, the protected frontage may be at significant risk of flooding. Of concern to both those who manage the structures and those who benefit from their shelter are the following questions:

- If these structures were to fail, to what height might they be reduced?

- b) How much wave protection would they provide in their collapsed form?

During the 2013 ICE Breakwaters Conference, the ICE Director of Knowledge Services (Mike Chrimes) challenged breakwater researchers to devise appropriate research through 'enabling' projects. The project presented here to answer questions a) and b) above was devised shortly afterwards, and funding for the suggested model studies was authorised by the ICE Research & Development Enabling Fund in October 2014.

Questions a) and b) were investigated in simplified 2-dimensional (2D) hydraulic model tests at a notional scale of 1:30, conducted to explore wave transmission across damaged and collapsed breakwaters for a set of example structures. To do so, however, this paper first reviews simplified structural analysis for blockwork walls; then scaling the test structures to represent example blockwork breakwaters and to reproduce a sufficiently realistic collapse process. This was achieved by:

- a) Review of historical documents for structural dimensions;
- b) An analytical wall stability model;
- c) Test-builds to ensure that the design was representative, but would collapse within the test facility;
- d) Examination of structural failure under wave action;
- e) Measuring wave reflections and transmission during and after the collapse process; and
- f) Evaluation of 'model effects' and their influence on the results.



Figure 5: Greve du Lecq, Jersey: (top) after collapse in 1879; (bottom) in 2014

[Sources: Jersey Harbourmaster; W. Allsop]

1.3. Outline of this paper

This paper starts by reviewing selected example breakwaters in Section 2, most of which have survived. (NB: It is particularly difficult to find details of failed structures as shown in Figure 5.) Section 3 describes the simplified analysis of block wall stability created to guide the design of the model tests. This section also presents results of the 'dry-build' tests to validate the spreadsheet model. The design of the main hydraulic tests are described in Section 4. The results of those tests are then separated, the collapse process is presented in Section 5, and the hydraulic results (reflections and wave transmission) in Section 6. Section 7 discusses the use of the wave transmission results in practice, and Section 8 offers a few remarks on further work.

Some of the results of this project were discussed in the 2017 ICE Breakwaters Conference by Allsop *et al* (2017) and Pearson & Allsop (2017).

2. EXAMPLE BREAKWATERS

The initial model design study reviewed breakwaters at Whitehaven, Blyth, Kilrush, Alderney / St. Catherine's, Peterhead, and Hartlepool. These structures are mostly vertical (or slightly battered) blockwork walls with random rubble infill; Peterhead is the exception, being 100% concrete blocks. Historical records contain few construction details, but nominal dimensions were extracted from photos, site visits, and old proceedings of the ICE. Key dimensions are summarised in Table 1. It is noted that core grading is generally not documented, but it is expected that the fill will be relatively evenly graded with an upper-limit diameter of a typical wall block, or perhaps smaller if the large blocks were used to protect the foundation mound.

It is probable that the strength of these breakwaters has relied primarily on the self-weight (and bonding) of the blocks in the walls, and on the stability of the foundation mound. Contributions to overall stability of any mortar used in construction will often be small, particularly where there has been little recurrent maintenance, and the mortar has degraded or been washed away over the years. Self-weight and friction between blocks will, therefore, govern stability. The distribution of gravity forces throughout the blocks will, however, be non-uniform. This irregularity of load transfer is an inherent feature of any blockwork, more so for mortar-less masonry, directly related to the precision (or otherwise) of block-cutting and laying tolerances. Additionally, abrasion of blocks and local settlement of the mound will redistribute and may reduce the forces which clamp the blocks in place.

Table 1: Dimensions of example blockwork breakwaters

Breakwater	Section			Lower blocks			Upper blocks		
	Height [m]	Width [m]	Block : Fill Volume [%]	Height [m]	Width [m]	Length [m]	Height [m]	Width [m]	Length [m]
St. Catherine's	18.0	12.0	28 : 72	0.8	1.0	1.5	0.4	0.6	0.8
Kilrush	8.9	14.4	36 : 64	-	-	-	0.45	0.7	1.0
Whitehaven	13.4	19.2	31: 69	2.2	-	3.3	-	-	-
Blyth	12.7	20.3	29 : 71	1.7	-	2.5	1.7	-	3.4
Heugh	4.0	10.0	-	-	-	-	0.55	0.5	2.1
Peterhead	15.9	14.0	100 : 0	2.0	2.3	3.9	2.0	2.3	4.1

The example, or generic, structures must be modelled so that both stability and collapse are realistic. This requires scaling of the blocks, core, and mound. The experiments were not set-up to test any particular structure, but to give test results that could be used to support generically applicable conclusions. A nominal scale of 1:30 was chosen to give a test section which could fit in the flume and could also be failed by test conditions within the range of the available equipment.

Blocks were manufactured in cement mortar to 140mm long, 70mm wide, and 30mm high with a (model) density of 2320 kg/m³. Correcting for the flume's fresh water versus seawater, these represent blocks at $\rho_c=2400$ kg/m³ of 4.2m long, 2.1m wide, and 0.9m high. These dimensions correspond well with Peterhead and Blyth, but are slightly large compared to the other example structures. These blocks were used for both 'dry-build' and wave flume 'wet' testing.

The model core material was widely graded with a nominal maximum diameter D_{max} of approximately 50mm. This was done to ensure that the maximum rock size was smaller than a wall block. Recalling that a typical core will be formed by discarded wall stones, chippings, and natural tout-venant rubble core, the natural upper-limit is indeed the wall-block size. This maximum size, therefore, corresponds to prototype D_{max} of approximately 1.5m, which is smaller than Peterhead and Blyth wall blocks. The minimum fill material size D_{min} was 5mm (0.15m prototype).

3. STABILITY OF SIMPLE BLOCKWORK WALLS

3.1. Structural stability of simple blockwork walls

Key structural failure modes are summarised below, given by Allsop (2009) which follows on from guidance in Bray & Tatham's *Old Waterfront Walls* (1992) and British Standard 6349-2 §7.3.1 (2010):

- sliding or overturning of a breakwater section as a single entity;
- global geotechnical failure of the foundation, destabilising the wall;
- removal of blocks from the wall causing discontinuity, thence structural instability; and
- local geotechnical failure of the mound, destabilising the wall and/or releasing fill.

Sliding or overturning as a single entity (a) and global geotechnical failure (b) have been relatively uncommon in the UK in recent years, but may have been significant earlier, whereas removal of blocks from the wall (c) and local geotechnical failure (d) are more frequent. The main failure mechanisms are, however, all governed by the self-weight of the blocks, friction between blocks, lateral earth pressure from the core, and hydraulic pressures acting across the blocks, individually and as an assembly. Waves striking the wall raise the phreatic surface within the core, and at different phases of the wave action will apply direct forces to the wall, positive or negative. As the waves recede, there is a temporary hydraulic gradient across the wall acting seaward. This increase in seaward pressure can cause the entire wall, or just a section, to collapse.

<i>Parameter</i>	<i>Unit</i>	<i>Magnitude</i>	<i>Sea</i>	<i>Blocks</i>	<i>Fill</i>	<i>Slide</i>	<i>Topple</i>
Number of blocks	No.	10	9	9.000	9		
Block length	m	4.2	8.1	8.100	8.1		
Block width	m	2.1		7.200			
Block height	m	0.9		6.300			
Still water level	m	8.0		5.400			
Wave height	m	0.0		4.500			
Block density	kg/m ³	2320		3.600			
Fill material density	kg/m ³	2670		2.700			
Block friction coefficient	-	0.7		1.800			
Fill porosity	-	0.35		0.900			
Friction angle	°	55					
Unit Weight of Water	kN/m ³	9.81					

Figure 6: Example input and output from simplified analytical model. Blue represents water level and saturated fill. Fill (not shown) is set to crest of top block. Red cell (bottom right) indicates instability.

The basic components being scaled, the next step was to design test structure height, width, and block configuration. The 2D test structure required a cross-section typical of the prototype structures to ensure realistic collapse mechanisms, but also needed to be constructible and stable within the confines of the flume. An analytical spreadsheet model and test builds were used together to investigate wall failure and guide design. The analytical model inputs block, fill, and water level characteristics to assess the stability of an arbitrary section of the wall, idealised as a single column. It assesses: a) self-weight of and friction between the blocks; b) hydrostatic and lateral earth pressures, including buoyancy effects; and c) sliding and overturning equilibrium of each block and set of blocks. This method of stability assessment is consistent with guidance by Bray & Tatham (1992) and British Standard 6349-2 (2010).

Example input and output are shown in Figure 6. The figure shows a cross section of the simplified wall; the blocks are grey, with open water on the left-hand 'sea-side' and saturated fill on the right-hand 'fill-side'. When the wall is built too high, or a large hydraulic gradient is applied, the wall becomes unstable, either through sliding or toppling of a section. When this occurs, a red cell is displayed at the height at which the instability occurred (e.g. at the toe of the rotating section).

The output from this simple model gives physically rational results. Higher wall heights reduce stability; wider blocks give greater overturning resistance; lower friction angles for the fill increase lateral pressure; and the buoyancy-effect on the wall blocks reduces resistance to sliding and overturning. These results were encouraging, but the model needed to be verified, particularly to clarify, remove or compensate for 3D effects from the flume sides.



Figure 7: Dry-build timber frame with partially constructed dry-build test.

3.2. Dry-build tests

The stability model discussed above can be used for either 'wet' or 'dry' conditions. For simplicity, a dedicated series of 'dry-build' tests using concrete blocks and granular fill (at nominal 1:30 scale) were, therefore, used to test the stability model. These dry-builds were contained in a timber 'box', as shown in Figure 7, which allowed testing the 2-dimensional assumption behind the analytical model. The characteristics of the blocks and fill used in the dry-builds were designed and measured to ensure that the analytical model inputs were 'tuned' correctly, so that a fair comparison could be made.

Key measurements were block density, fill density (rock density and fill porosity), inter-block friction coefficient (pull-tests), and friction angle (estimated through angle of repose tests). The first angle of repose measurements were completed with a core that had been left to dry for some weeks. A 'wetted' state test was carried out where water was poured into the mix until it was running out of the material. Finally a 'saturated surface dry state' (SSD) with moisture content between the first and second tests was carried out once the core had dried again. The results were all surprisingly close and suggested $\phi = 53 - 55^\circ$ to give reasonable estimates of the angle of repose of the fill.

The dry-builds were constructed one row at a time, slowly and carefully placing the fill as the walls got higher, until collapse. Initial dry-builds showed significantly higher stability than predicted by the analytical model; 20 rows high at failure in the dry-builds versus 11 or 12 in the model. This increased stability was attributed largely to jamming, or arching against the flume walls. As the blockwork wall grew higher, the centre of the wall started to bow, arching across the frame, pushing edge blocks outwards and against the side walls of the frame. The stiff timber frame provided lateral support, which pinched the blocks together and gave the wall greater strength against overturning. Arching of a wall in plan by up to 50mm (1.5m prototype) at the centre of the crest was observed. Once this was exceeded, the arch would snap through suddenly, the wall would unzip along the vertical centre line, and the structure would collapse entirely. A typical failed dry-build is shown in the left-frame of Figure 8, accompanied on the right by the same failed structure with the spilled core material dug out. This reveals the arched, but still intact, toe of the wall, about which the upper section toppled. It shows that the wall overturned with its hinge located not at the bottom row, but a few rows up.



Figure 8: Failed dry-build. Left - immediately after collapse. Right - failed structure with spilled core removed to reveal arched (but still intact) wall toe.

This contrasts the spreadsheet model which always showed rotation about the lowest row. The difference of behaviour in the dry-builds may be, in part, due to the arching effect providing additional strength to the wall before a brittle collapse, but also perhaps due to the stability model's assumption of simple Mohr-Coulomb lateral earth pressure. This theory assumes that the force applied by the fill is horizontal, whereas strictly it acts slightly downwards and diagonally, activating friction between the fill and the wall.

The downward frictional component of this pressure tends to stabilise the wall by clamping the lower blocks more tightly into place, reducing the overturning, or disturbing, moment. This effect was noted by Bray & Tatham (1992) and is illustrated in Figure 9 from their book. It is noted that for an internal friction angle of $\phi=25^\circ$ and no wall friction angle ($\delta=0$), the disturbing moment is equal to the restoring moment. When wall friction is increased to $\delta=\phi$, the restoring moment becomes 2.3 times the disturbing moment. It is uncertain whether the full effect of wall friction is activated on the scale of the dry-builds, but its omission in the analytical model along with the arching effect may explain partially why the simple model under-predicts wall stability and does not show toppling over a toe section.

In an attempt to reduce the arching effect across the wall, low-friction PVC plates were installed on the inside faces of the timber frame to reduce friction between the blocks and the rough plywood sides. This reduced arching and brought the stability of the wall down to a closer agreement with the analytical model: 12-15 rows high at failure in the dry-builds versus 11 rows in the analytical model.

Tests were also performed with the blocks resting on their narrowest face, or 'on-edge'. This reduced the stable wall height in the dry-builds by approximately 75-80%, from 12-15 blocks high to 3 blocks high. This sharp reduction in wall height matched results from the analytical model. The dry-build tests showed that the analytical model under-predicts wall stability by approximately 25-30%, even with PVC slip plates installed. This may be due largely to arching across the wall, which can be reduced in the model, but not eliminated. It was noted that this arching effect would likely be worse in the flume, where the walls are made of stiffer concrete which would provide more rigid lateral support. The dry-build tests did, however, confirm that a test structure could be built to at least 15 rows high. With a block height of 30mm and a geometric scale of 1:30, this corresponds to a prototype wall height of 13.5m, which matches well with the heights of typical blockwork breakwaters.

Another difference between the analytical and physical models is in the 'perfection' of the blocks. All the analytical blocks were perfectly rectangular with smooth faces, without any rounding at the edges. The physical model blocks were slightly variable, and by handling through these experiments, edges were abraded and, therefore, slightly rounded. The effective rotation points will, therefore, have moved inward, reducing the restoring lever arms.

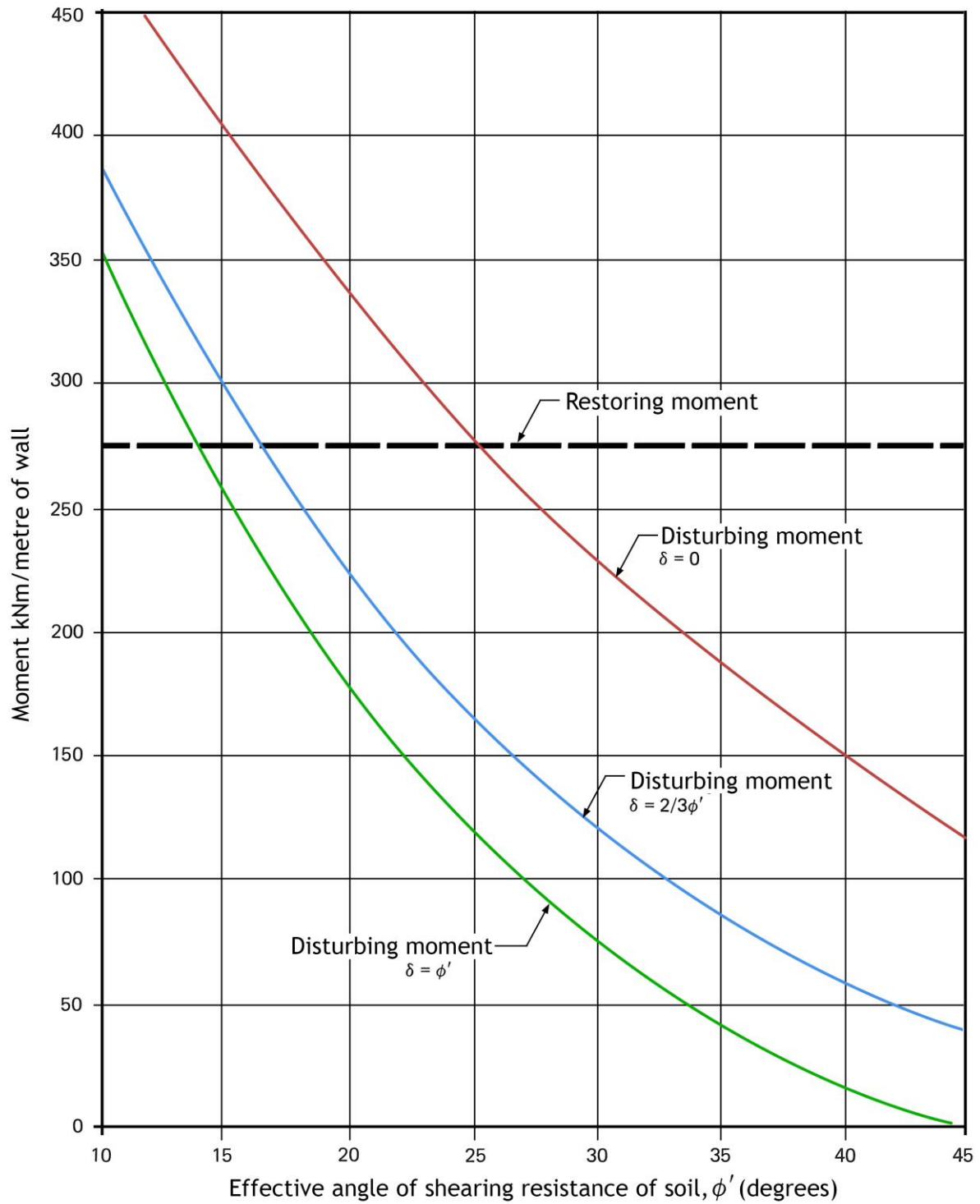


Figure 9: Effect of internal friction angle on overturning (disturbing) moment. Adapted by Adrian Pearson from Bray & Tatham (1992).

4. DESIGN OF THE MODEL TESTS

The main (wet) physical model tests were designed primarily to identify how much (or little!) wave protection might be given by the remains of such a breakwater after collapse. The main issues are therefore: to what crest level might it be reduced after initial failure; and how large might the transmitted waves be? To do this, the wave tests were designed to collapse the walls as realistically as possible (given the simplifications to block sizes etc. discussed in section 2. The tests would then measure wave transmission – the key output.

Tests in the flume were organised by structure and number. One complete set of tests on a particular structure is called a “Series”. Each Series comprised different test parts, progressing from Test Part 1 – the first waves – through to the final test where the collapsed breakwater reached a stable form. A summary is shown in Table 2 and a full description of series and test parts is given by Allsop et al (2017). An example test structure in the flume is shown in Figure 12.

4.1. Wave conditions

In the initial Test Parts, storm waves at a steepness of $s \sim 0.06$ were generally used to fail the structure. Then once collapse had been initiated (or for most cases – completed), subsequent Test Parts used Persistent waves ($s \sim 0.035$), and then Swell waves ($s \sim 0.01$) to measure transmitted (and reflected) waves.

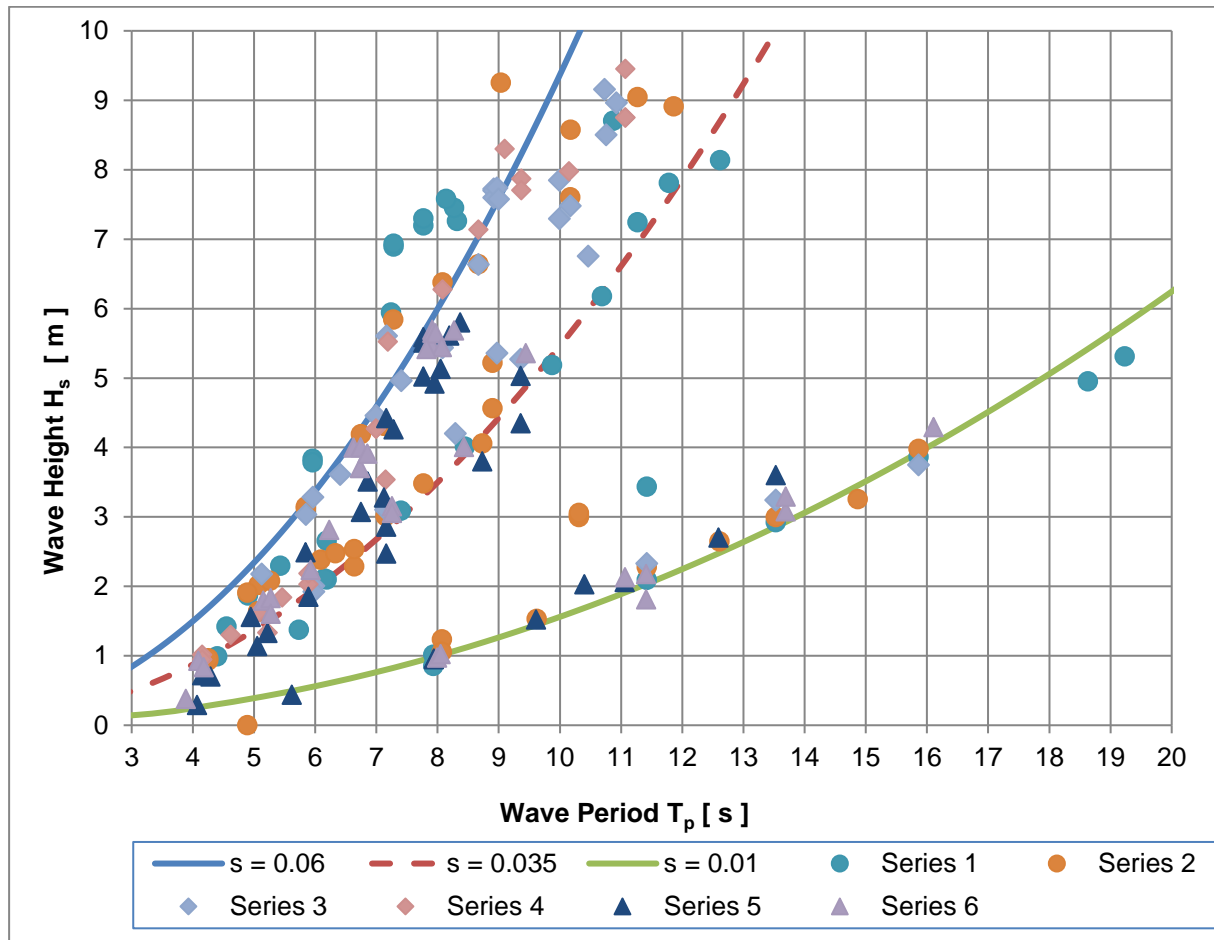


Figure 10: Test conditions, Storm ($s=0.06$), Persistent ($s=0.035$) and Swell ($s=0.01$)

Wave conditions at the chosen 1:30 scale are illustrated in Figure 10, extending up to $H_s \sim 9.0\text{m}$ (at 1:30 scale), typical of large North Sea storms. Three wave regimes were used, identified by their target wave steepness: Storm ($s \sim 0.06$), Persistent ($s \sim 0.035$) and Swell ($s \sim 0.01$).

In each Test Series (see Table 2), wave conditions started at $H_s \sim 1.0\text{m}$, and were stepped up in each Test Part, most of which were run for 500 (or a few for 1000) waves. The shorter duration was generally used for the initial wall failure tests using waves of $s \sim 0.06$. Use of the shorter duration was based on testing of wave overtopping by Reis *et al* (2008), and Romano *et al* (2015), whose studies suggested that 500 waves may be sufficient to give good representation of both mean and maximum effects, even for highly non-linear responses such as peak wave overtopping volumes.

4.2. Test structures

The walls all used concrete blocks equivalent to $4.2\text{m} \times 2.1\text{m} \times 0.9\text{m}$ and density equivalent in seawater to $\rho_c = 2.4\text{t/m}^3$. In most of these tests, the blocks were laid in a single skin with the long side across the test flume, and in height steps (rows or courses) of 0.9m . A few of the dry-build tests explored the stability of blocks on edge.

Table 2: Summary of structures, Test Series and Test Parts					
Test Series	Rows	Wall height / Crest	Test Parts	Wave heights	Wave Type
1	16	15.8m unarmoured	1 - 14	1.4 – 8.7m	Storm, Persistent
			15 - 24	1.0 - 8.1m	Persistent
			25 - 36	0.8 – 5.3m	Swell
2	18	17.45m unarmoured	37 - 52	1.4 – 9.2m	Storm
			53 - 60	0.9 – 4.1m	Persistent
			61 - 80	1.1 – 5.2m	Swell (66 – 80, others)
3	16	16.23 paved crest	81 - 100	2.2 – 9.2m	Storm
			101 - 110	1.9 – 5.3m	All types
4	20	18.8 unarmoured	113 - 122	4.3 – 9.4m	Storm
			123 - 135	0.9 – 4.9m	Persistent
5	11 + 2	12.2 paved	136 – 144	1.3 – 5.8m	Storm
			145 - 169	0.3 – 5.1m	All types
6	11 + 2	12.2 paved	170 - 181	0.4 – 5.7m	Storm
			182 - 185	0.9 – 4.0m	Persistent
			186 - 200	1.0 – 5.4m	All types

The preliminary dimensions of the test structure cross-section were $\sim 13.5\text{m}$ high, 30m deep (front to back) and 30m across the flume. The 'typical' ratio of blocks to fill was 35% to 65%. The block sizes

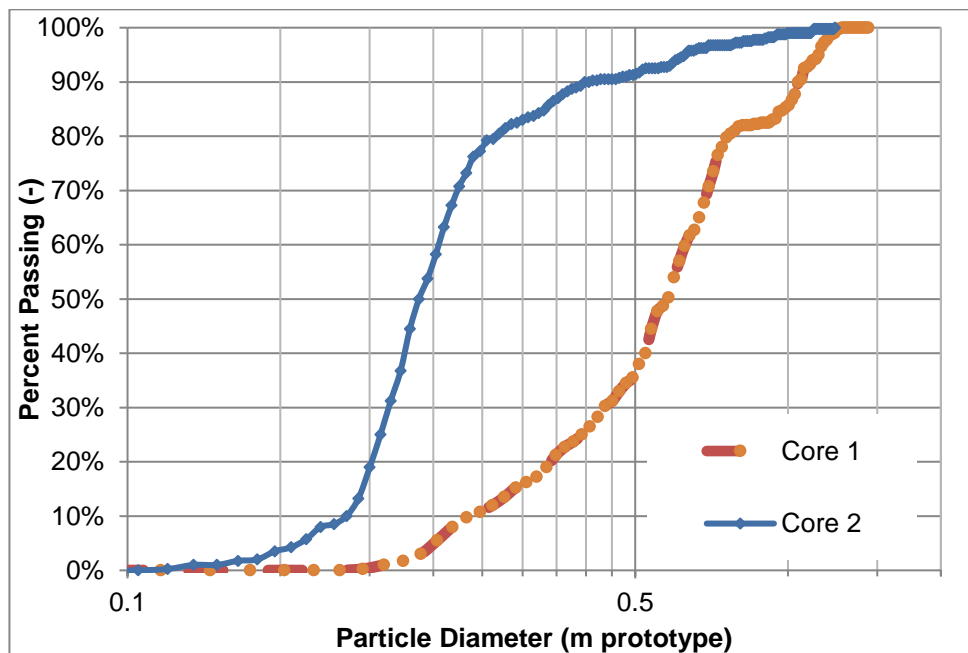


Figure 11: Rubble core samples, original (core 1) and finer (core 2).

correspond well with those at Peterhead (Figure 4) and moderately with those at Blyth (Figure 3). The smaller blocks in the other breakwaters reviewed were less well represented by these large block sizes.

The model core material was widely graded with $D_{\max} \sim 1.5\text{m}$ (Figure 11), to ensure that the maximum rock size was smaller than a typical wall block. Recalling that a typical core will be formed by discarded wall stones, chippings, and natural tout-venant ('all-in') rubble core, the natural upper-limit is indeed the wall-block size, smaller than Peterhead and Blyth wall blocks. For the final Test series 6, a finer core (core 2) was used to explore its effects on the collapse process.

In the wave flume, all blockwork walls were constructed on a rubble mound from flume floor at -24m to a platform level nominally at -10m, see example in Figure 12. All levels are expressed relative to nominal still water level of 0.0m.

The test programme was formed by six Test Series, each run on slightly different structures. The early test structures suffered from excessive levels of restraint from the side walls, delaying the onset and process of wall collapse. It is not, however, expected that those effects will have significantly impacted on the wave transmission results.

5. RESULTS – COLLAPSE PROCESS

Tests in the flume were organised by structure and number, with a complete set of tests on a particular structure called a "Series", see Table 2. Each Series included different test parts, progressing from Test Part 1, through the test where the collapsed breakwater reached a stable form. Thereafter the tests continued to measure wave transmission over the collapsed profile. The test programme is summarised in Table 2. An example test structure in the flume is shown in Figure 12.

5.1. Test observations

Series 1

The test structure in Series 1 was constructed 16 blocks high, both front and back, bonded pattern. To limit mixing of the wall fill with the rubble foundation mound, a geotextile was laid over the mound. This proved to be of limited utility and was omitted in later tests. Starting with storm waves at $H_s=1.4\text{m}$, the front wall only began to fail when waves had increased to $\sim H_s=7.2\text{m}$, when the crest course was knocked off, pushing the underlying blocks backwards. Uplift pressures then lifted crest blocks out of the wall. The reduction in crest level then led to increased local overtopping, in turn washing out fill material.

Multiple blocks were extracted seawards from the front face as overtopping and local wave pressures opened multiple gaps in the blockwork at a high level. Some courses resisted further due to arching of the blocks against the sides of the flume, clearly a model effect. As more blocks were extracted, more overtopping penetrated into the rubble fill, forcing out further blocks. Once the front wall had failed at about $H_s=7.2\text{m}$, wave backwash steadily washed fill out until the seaward blocks and rubble fill were graded to a relatively shallow slope angle down the front of the structure. With the front wall down, waves broke onto the new rubble beach, attacking the rear wall significantly less. The rear wall itself again showed considerable arching, probably leading to the wall staying intact to $H_s=8.7\text{m}$, much longer than might be expected without lateral restraint. Photographs of this progressive type failure are shown in Figure 13. Measurements of front and rear wall crest heights are shown in Figure 14. It is noted that crest level is defined as the highest point of the structure. After the rear wall failed, the core may often have been higher than the remaining wall sections.



Figure 12: Example test structure in flume.



Figure 13: Progressive failure of Series 1.

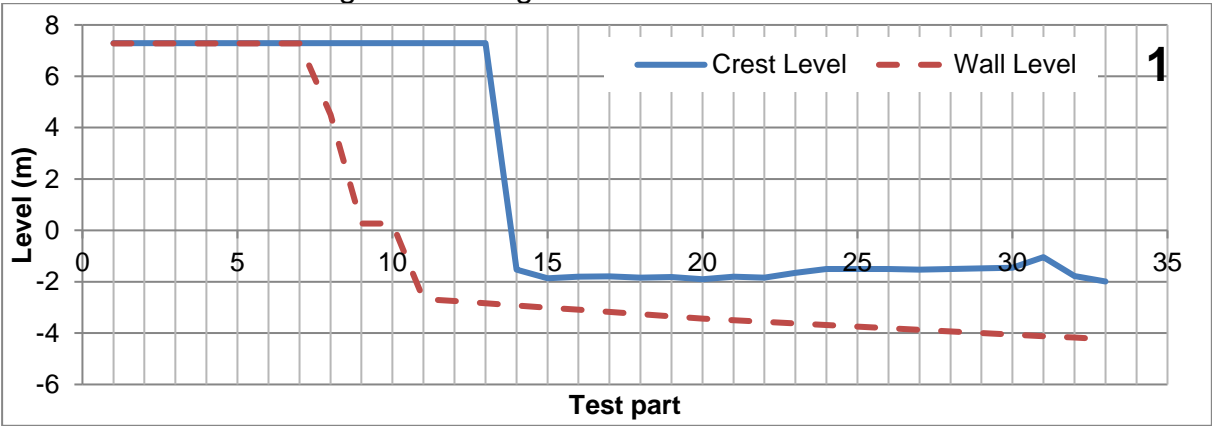


Figure 14: Progression of Test Series 1, test parts 1-33

Series 2

The wall in Series 2 was constructed 18 blocks high on the front, but fill and back wall were taken only to 16 courses. Blocks were again laid in bond. The geotextile in Series 1 was omitted as it was not useful. Sixteen test parts (each 500 waves) were required, failure occurring over test parts 48-52 ($H_s=8.6-8.9\text{m}$), illustrated in Figure 15.

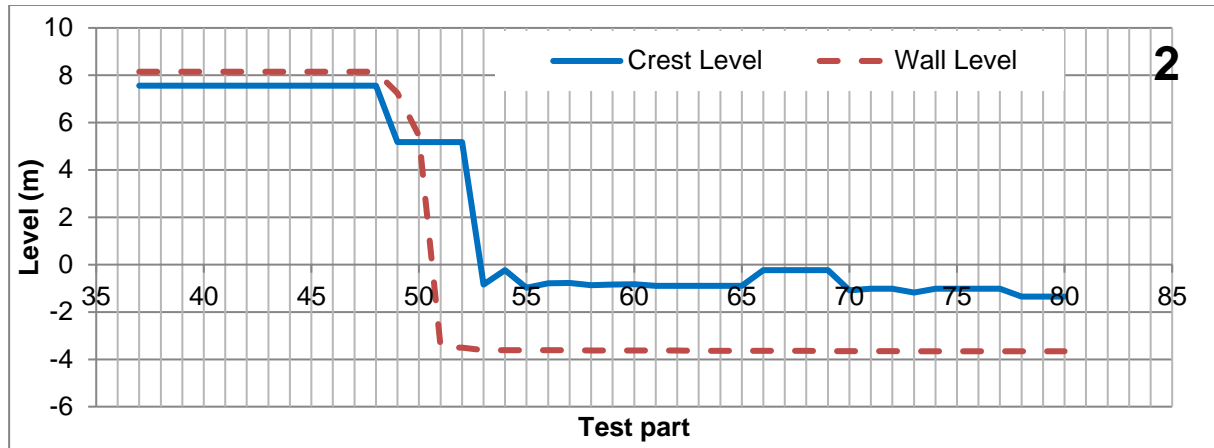


Figure 15: Progression of Test Series 2, test parts 37-80.

Wave conditions were increased to $H_s=8.6\text{m}$ when some of the top course were flipped back by overtopping waves. As the test progressed, around 3 blocks were 'jerked' forward, allowing upward flows to lift and flip them backwards, as in Series 1. Once this key group of blocks had been removed, waves scoured out the remaining blocks and fill, exposing the rear wall to direct wave impact.

Again the speed of wall collapse was probably delayed by arching effects giving artificial restraint. As seen before, the front wall mostly collapsed seaward. The remaining courses of toe blocks acted to reduce erosion of fill material, a process that has been seen on site for collapsed walls. Measurements of rear wall and crest heights are shown in Figure 15. The eroded profiles (see Figure 16) were very similar to those seen for Series 1.



Figure 16: Post collapse view of Series 2, seaward side to right. Note the intact remnant of the wall toe.

Series 3

The wall in Series 3 was built 15 blocks high in columnar bond with standard fill. As this was rather lower than Series 2, it was expected that overtopping damage might be greater. A blockwork 'capping' layer on fines brought the section height to 16 blocks all the way across, see Figure 17.

With the lower initial crest level, the wall in Series 3 did indeed suffer increased overtopping. At $H_s=7.05\text{m}$, the front edge of the capping layer began to lift due to the up-rushing waves. This caused the crest protection to deform, continuing until the row had been either thrown backwards onto the structure or washed forwards creating a gap in the front of the wall. Once this gap had been opened, waves were able to penetrate under the cap, washing individual blocks off relatively quickly. This allowed waves to pull the fill and front wall down within 500 waves, forming a profile similar to those seen in Series 1 and 2. Measurements of rear wall and crest heights are shown in Figure 18.



Figure 17: Series 3 with protected crest.

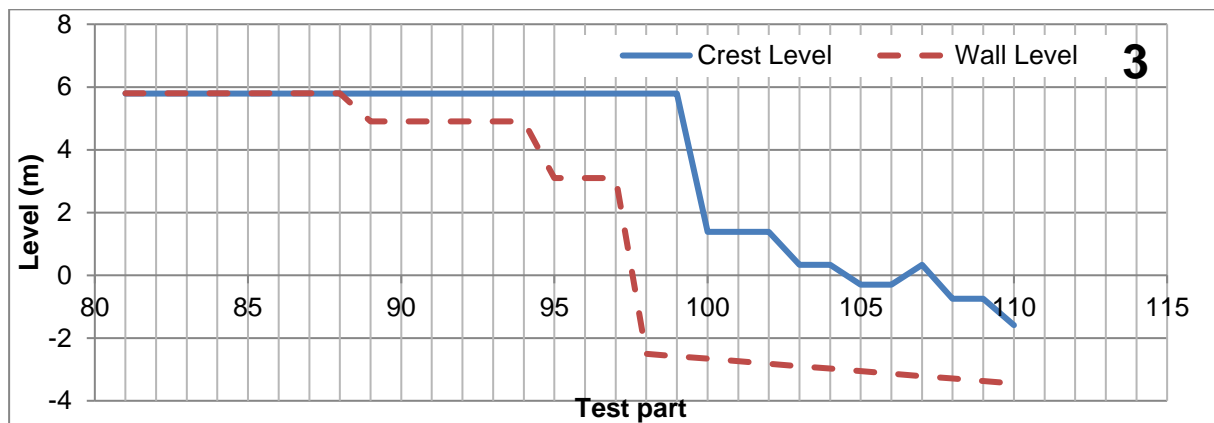


Figure 18: Progression of Test Series 3, test parts 82-110

Series 4

The structure for Series 4 was built to a height of 20 blocks in columnar pattern, with PVC side panels to reduce end restraint. At $H_s=6\text{m}$ the top layer began to show movement, with some blocks pushed backwards and one in the second row being extracted forwards. Soon more blocks were extracted from the central columns, one or two rows down, allowing fill to be extracted and creating a sink-hole behind the inside face of the front wall. Once enough blocks had been extracted, a section of wall began tilting backwards, and some blocks then fell down into gaps where those had been extracted. This drop in height allowed waves to overtop and wash out loose blocks and fill.

The following tests ($H_s=8\text{m}$) steadily brought the front wall down course by course, washing the fill out with it. The rear wall failed in a more brittle mode, perhaps due to reduced arching in this build. More core was washed over into the rear face when it collapsed, and the rear wall collapsed more as a single unit in Series 4 than in Series 3. Measurements of rear wall and crest heights are shown in Figure 19.

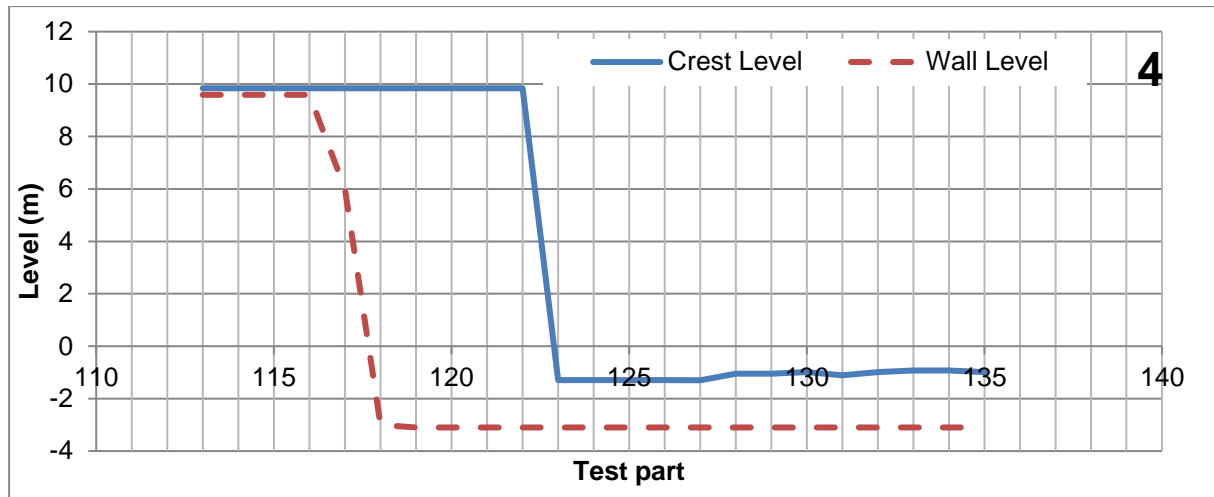


Figure 19: Progression of Test Series 4, test parts 113-135

Series 5

The walls in Series 5 were constructed only 11 blocks high, giving a crest level at +4.4m, and a wall width of 24.45m. Following experience in Series 3, the reduced crest level was expected to overtop more so the crest was protected by 3m x 3m aluminium slabs ($\rho_c \sim 2.4\text{t/m}^3$) equivalent to 0.1m thick. As common in breakwaters of this era, a parapet wall along the front edge was formed by two rows of wall blocks placed end-on; see Figure 20. Additional 'slip' measures were taken to reduce any artificial restraint to the blocks given by the sides of the wave flume.

Alongside the reduced crest level, a further change for Series 5 was to reduce the test water level by 1.5m to a nominal level of -1.5m. As before, wave conditions were ramped up at the 'storm' steepness ($s \approx 0.06$). In Test 138 at $H_s = 4.3\text{m}$, some parapet wall blocks started to slide backwards. During Test 139 at $H_s = 4.9\text{m}$ more crest blocks were pushed backwards, thus allowing wave forces to lift the parapet wall blocks. During Test 141 ($H_s = 5.8\text{m}$) half of the front wall collapsed around the hole left by two blocks extracted previously. During Test 142 (at essentially the same wave condition) the remaining half of the front wall collapsed. Most capping slabs were swept over the crest onto the rear foundation. The rear wall collapsed during Tests 143 – 145 ($H_s \sim 5.5\text{m}$). Measurements of rear wall and crest heights are shown in Figure 21.



Figure 20: Test section for Series 5, showing crest slabs and parapet wall

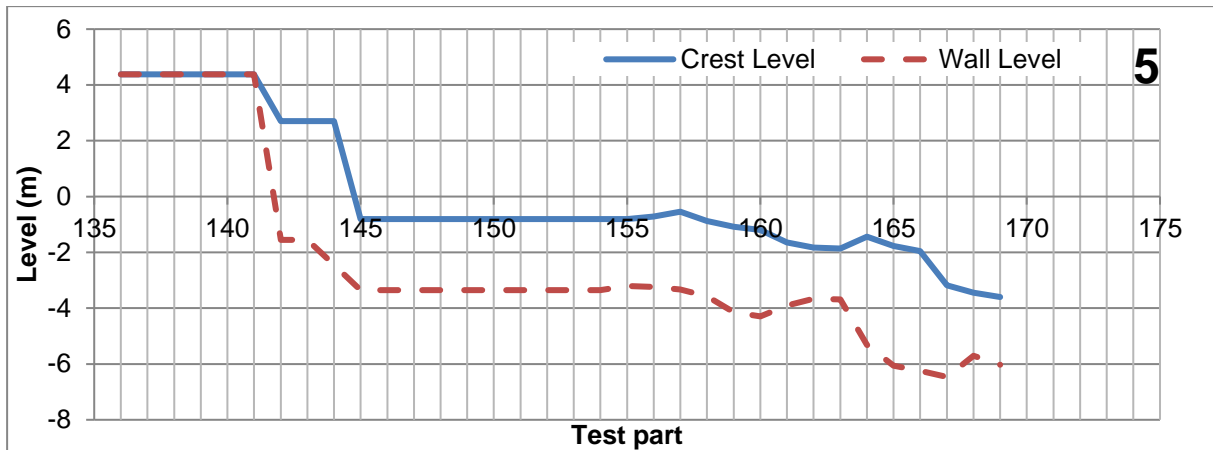


Figure 21: Progression of Test Series 5, test parts 136- 169

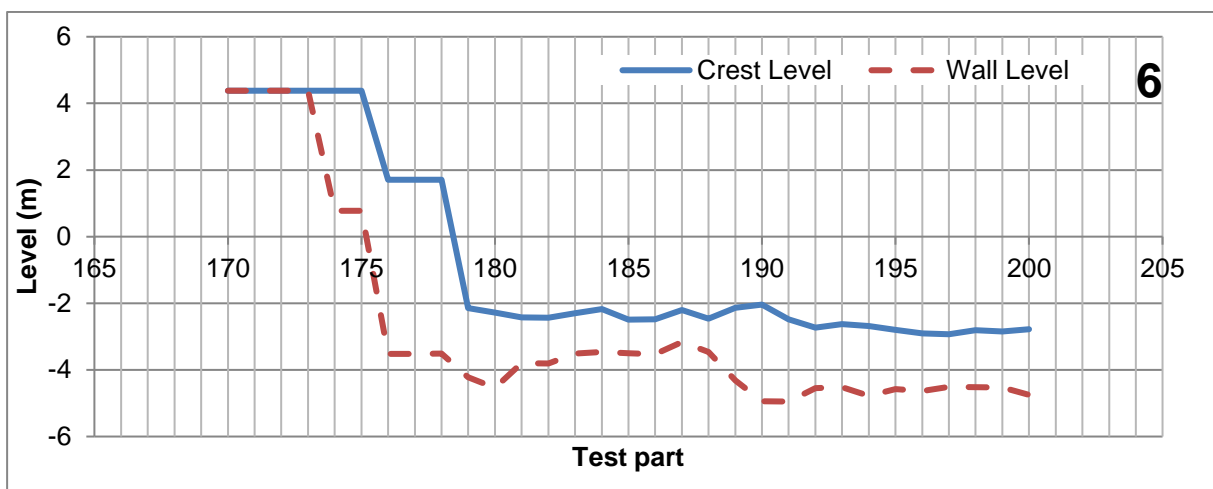


Figure 22: Progression of Test Series 6, test parts 170-200

Series 6

The test section in Series 6 was very similar to that in Series 5, again with main walls of 11 rows, a protected crest using large slabs, and a parapet wall of two layers of blocks placed end-on, except for the replacement of fill by the finer Core 2, see Figure 11.

Waves in Series 6 were increased from $H_s=2\text{m}$ to $H_s=5.7\text{m}$ when the first block was extracted from the front face, and crest protection slabs began to be washed off. Further tests at $H_s=5.7\text{m}$ steadily extracted more blocks, particularly in rows just below the capping layer. During Test 173 a row of blocks was extracted and the parapet wall was pushed back so that it rested mainly on the crest slabs, rather than on the main front wall blocks. Further waves pulled blocks out seaward, causing a large hole in the centre of the section. This collapsed quite quickly, and the front wall was pulled over during Test 177 ($H_s=5.5\text{m}$). The rear wall failed faster than in previous tests, probably due to reduced arching. The top blocks were pushed back, then core material filled around them to form a fairly stable slope; see Figure 22. Figure 23 below shows the typical evolution of collapsed profiles.

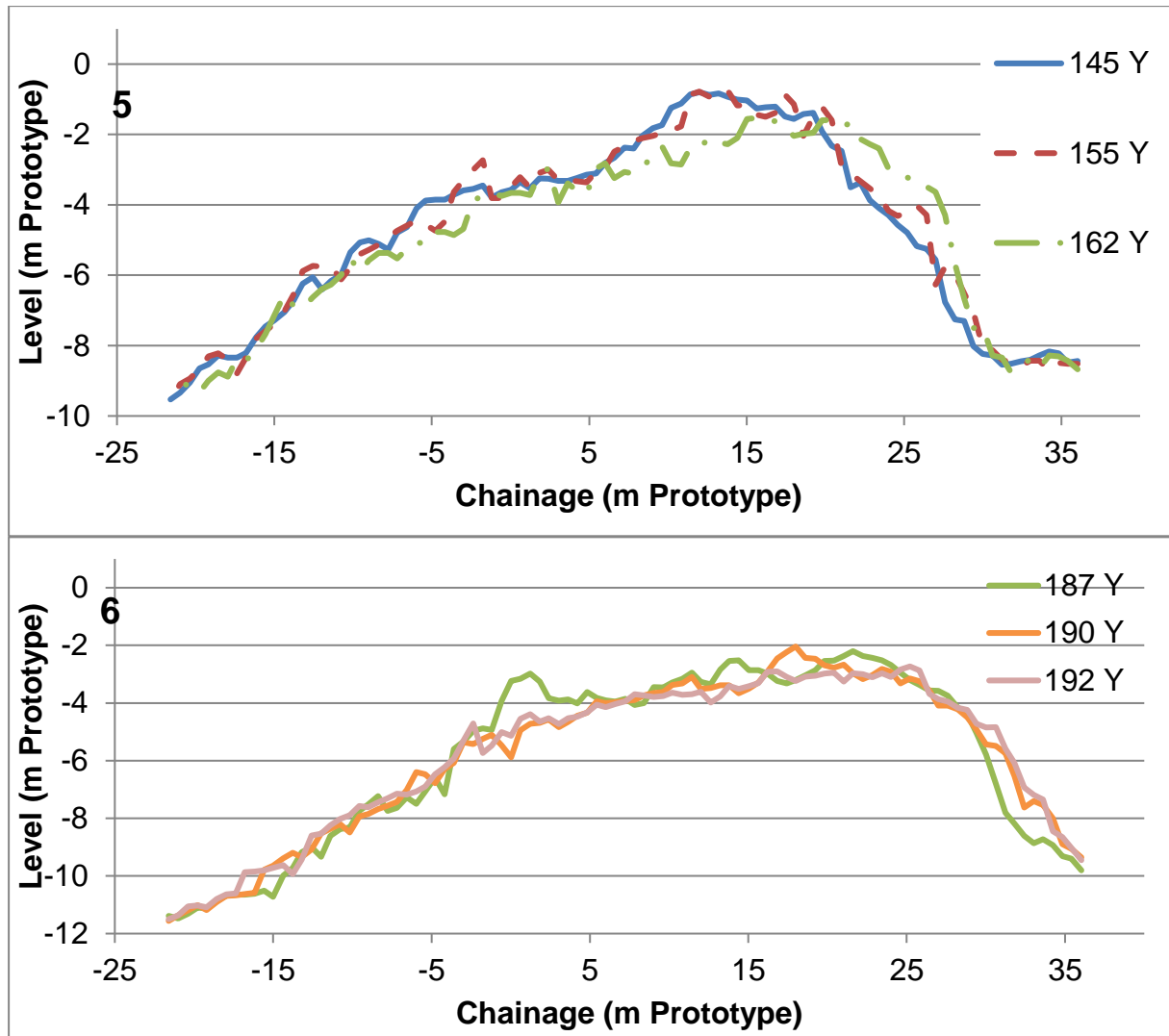


Figure 23: Typical evolution of collapsed profiles, Test Series 5 and 6.

5.2. Discussion of results

The review of historical documents, photos, and site visits gave 'typical' sections of old blockwork breakwaters. Section dimensions, block sizes, and core grading were scaled down and, with the use of the analytical model, enabled the initial design of a test structure. The model, although limited slightly by its simplifications, provided insight into wall failure mechanics once compared to dry-build tests in a timber frame. The comparison showed that 3-dimensional effects, specifically arching across the wall, lent significant extra strength to the wall. PVC slip plates were installed between the edge blocks and the frame walls, decreasing the friction between the two surfaces. This reduced arching and gave better agreement between the model and dry-build tests. These two experimental methods together aided the design of the test structure for the flume.

The flume tests showed higher wall stability than the dry builds, perhaps due to the rigid flume walls providing strong lateral support and enhancing the arching effect. PVC slip-plates alleviated this effect, but not entirely. Failure was less brittle and more progressive. Upper courses of blocks were jarred loose by wave impact, typically at $H_s = 7-9\text{m}$, and then overtopping and downfall pressures extracted them fully. This exposed the core to direct wave attack, accelerating the front wall failure. The rear wall showed more stability than expected, again due to arching. Once the walls had failed, the remains acted as a 'non-engineered' rubble mound with wall blocks scattered across the front and rear slopes, and visible remnants of intact wall toes. Despite the simplified geometry of the structures and the lack of degradation and settlement effects, the test structures in the flume showed encouragingly sensible failure mechanisms.

One unexpected result of the dry-build tests was that the hinge of rotation was some 3-4 rows above the toe of the wall, see especially Figure 16. An explanation has been derived from the effect of friction between the fill and wall in Chapter 3. Supporting evidence is also given by the collapsed breakwater at Skateraw (near North Berwick) shown in Figure 23. This breakwater had been constructed 1799 -1825 by two farmers, Brodie of Thorntonloch and Lee of Skateraw, to import coal, and export limestone and potatoes, see Graham (1969). The 85m long breakwater / pier was probably destroyed 1853 -1892.



Figure 23: Remains of breakwater at Skateraw, probably collapsed between 1853 – 1893, still showing remains of wall foundation

6. RESULTS – WAVE TRANSMISSION AND REFLECTIONS

Each Test Series used wave conditions following the three mean wave steepnesses in Figure 5. Wave gauges seaward of the test section quantified incident and reflected waves, and another behind measured transmitted waves. Reflected and transmitted wave heights were then divided by the incident height to give reflection and transmission coefficients C_r and C_t , respectively. Values of C_r and C_t have been plotted below series by series to give indicative 'histories' of each test. These graphs may be read with the 'damage progression' graphs given in Chapter 5.

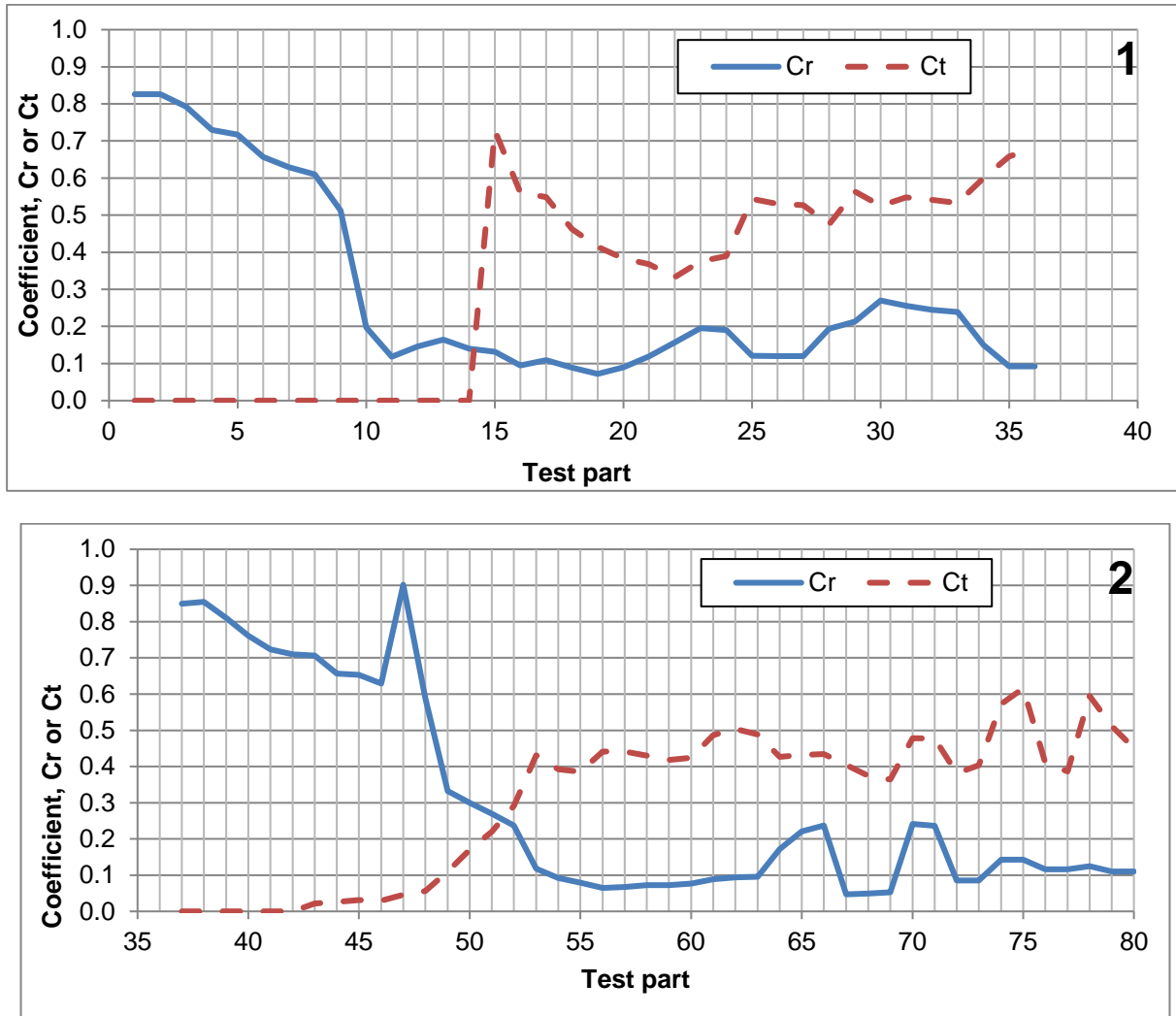


Figure 24: Reflections and transmission, Series 1 (top) and Series 2 (bottom).

Series 1

For Series 1, the reductions in C_r in Figure 24 suggest that damage to the front wall started relatively early. Conversely, wave transmission shown by C_t depends strongly on failure of the rear wall. The sudden increase of transmission in Test Part 14, therefore, correlates well with observations of the rear wall failure. Thereafter, reflections continue at a relatively low level, $C_r < 0.25$, whilst transmission stabilised at $C_t \sim 0.55$.

Series 2

In Series 2 (also Figure 24) the process of wall failure was more abrupt with increased transmissions around Test Part 52 following soon after the fall in reflections as the rubble slope develops from the debris of the front wall collapse in Test Part 49.

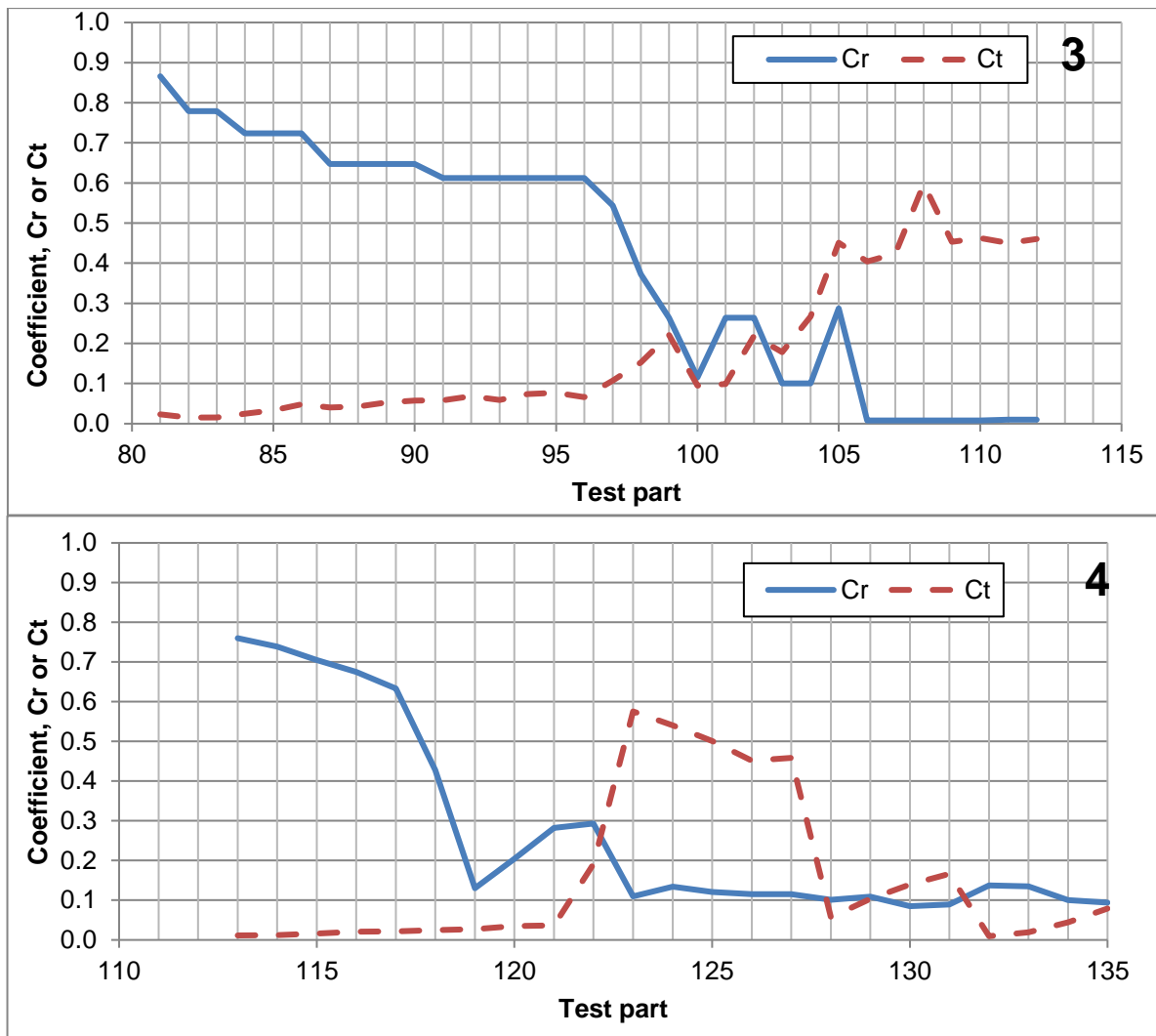


Figure 25: Reflections and transmission, Series 3 (top) and Series 4 (bottom).

Series 3 and 4

The wall failure in Series 3 was more gradual with reflections reducing as waves overtopped more (Figure 25). The rear wall failed incrementally, with transmission increasing as reflections reduced. This process was repeated in Series 4 (also in Figure 25), again with a lag in lower reflections as the front wall fails before the rear wall.

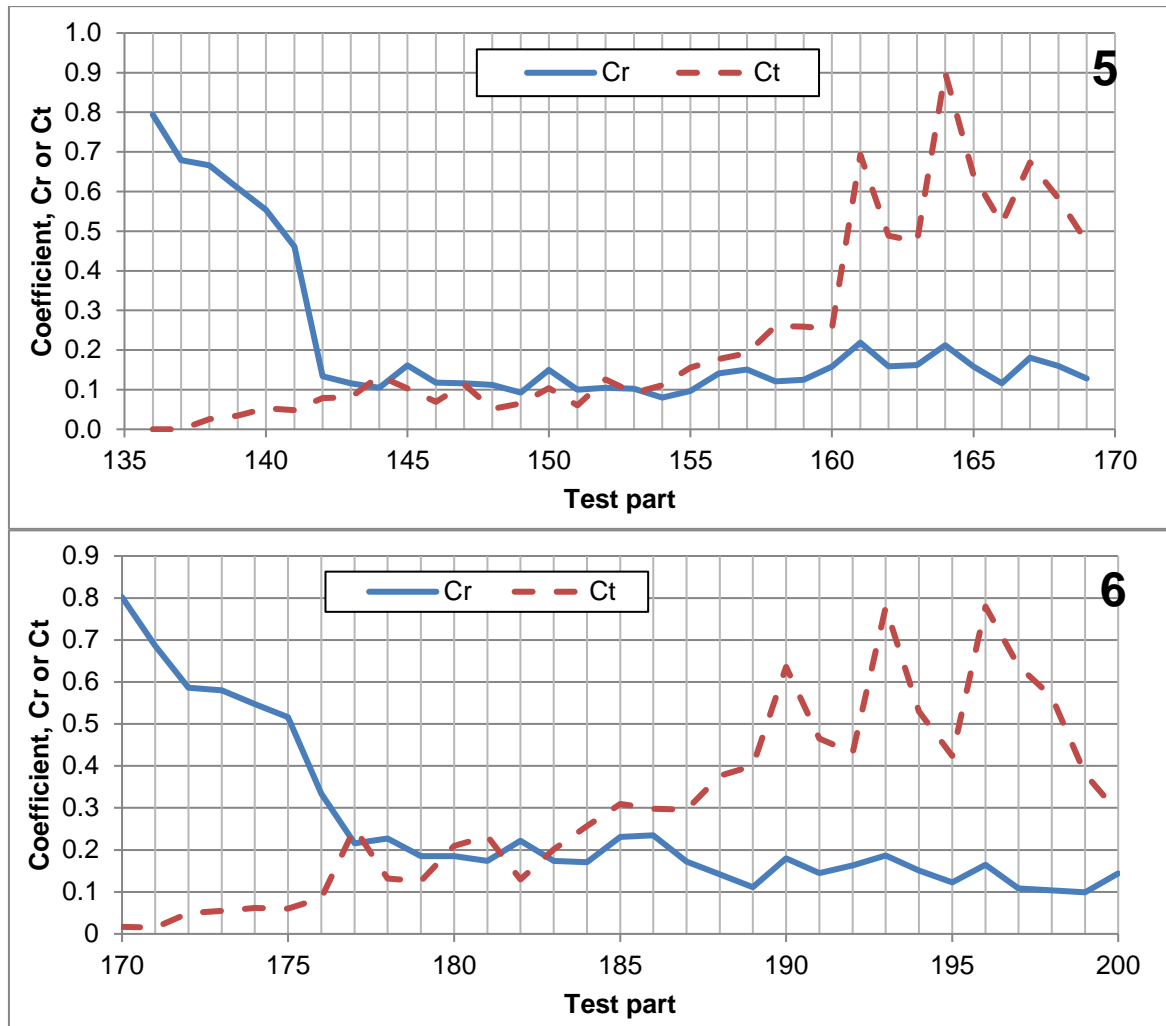


Figure 26: Reflections and transmission, Series 5 (top) and Series 6 (bottom).

Series 5 and 6.

As expected, Series 5 and 6 behaved similarly. Reflections (Figure 26) fell rapidly with increased overtopping, then as the vertical front face degraded to a slope. Again the increase of transmission (also Figure 26) was delayed until the rear wall collapsed, then varying with the test wave condition.

7. DISCUSSION ON USE OF THESE RESULTS

The form of presentation of wave transmission measurements in section 6 links C_r and C_t to the damage process, but this form is not easily used to predict transmission. Values of transmission coefficient C_t have been plotted against the simplest dimensionless freeboard, R_o/H_{si} , in Figure 28, using mound crest levels from the profile measurements and the test water level to calculate R_o . Results for each of the test steepnesses have been plotted separately in Figure 27. An attempt was made to explore whether an alternative approach by Powell & Allsop reduces scatter by including wave steepness where C_t is plotted against $R^* = (R_o/H_{si})/(\sqrt{s(2\pi)})$. The improvement was slight and judged not worth the increase of complexity in the plotting parameter.

As presented in Figure 27, the results show that there were very few instances when the crest of the damaged mound fell below $R_o/H_s \sim -2$, or where wave transmission exceeded $C_t \sim 0.6$. The few instances of higher C_t arose from inherently smaller wave conditions. At first pass therefore, these results suggest that these two limits might safely be used in any initial assessment.

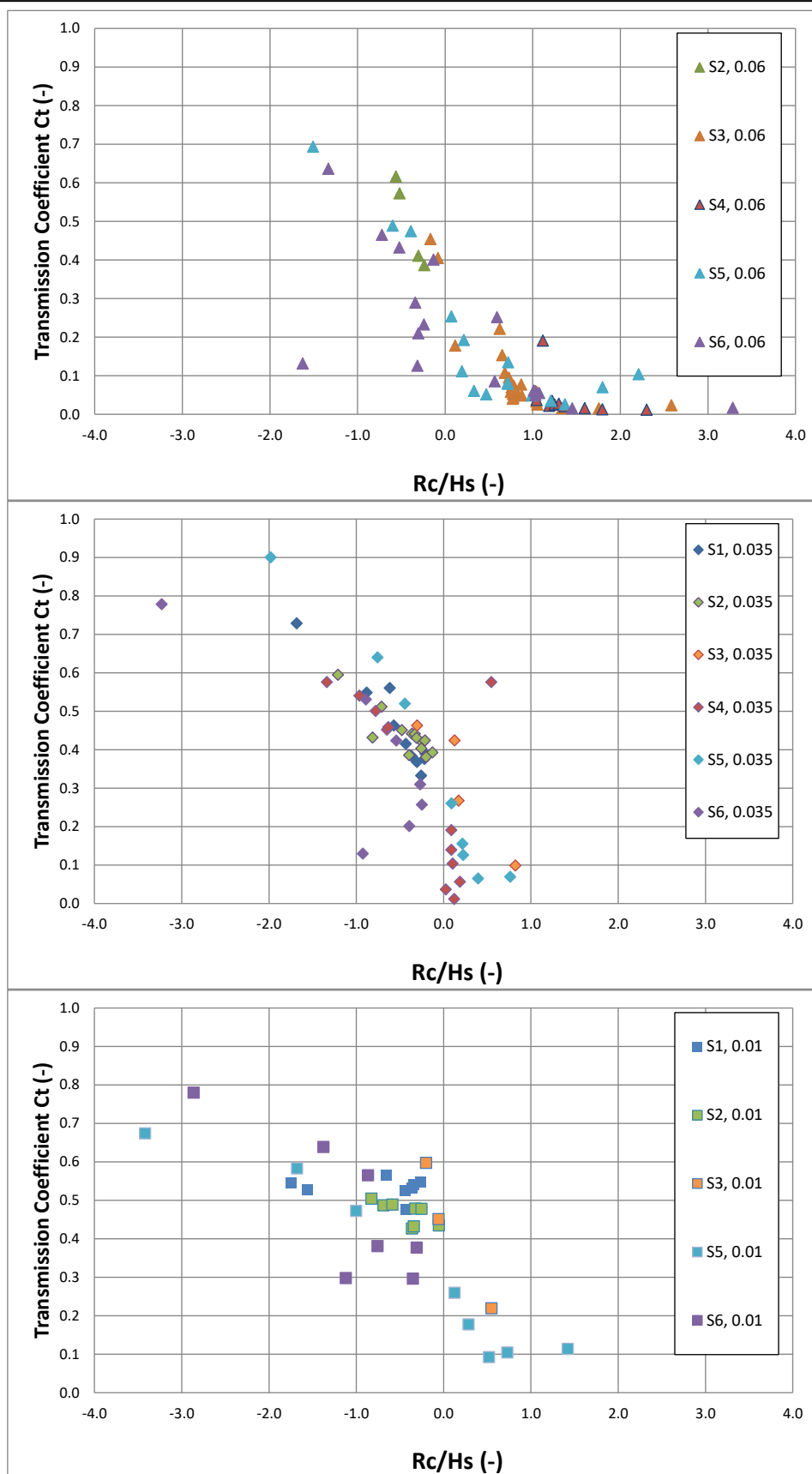


Figure 27: Wave transmission, C_t vs R_c/H_{si} : $s = 0.06$; $s = 0.035$; $s = 0.01$

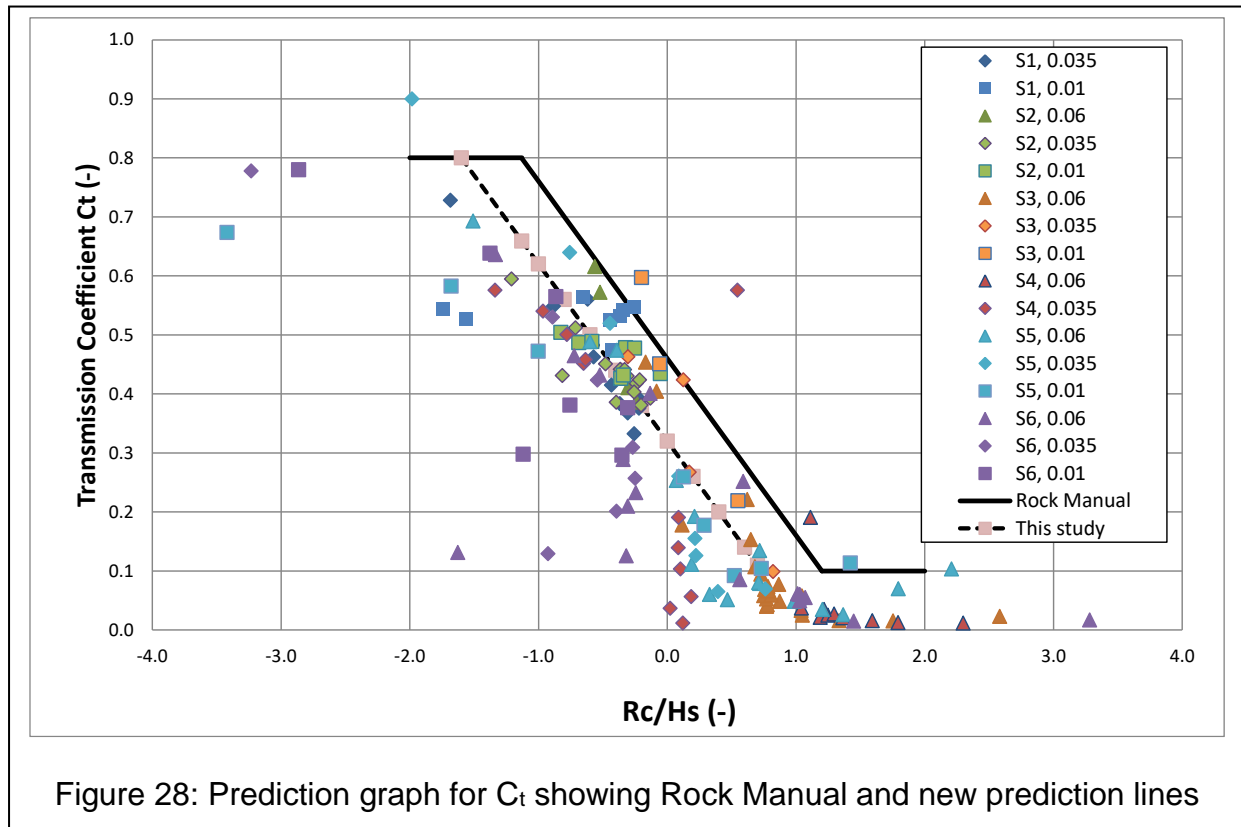
The results in Figure 27 do not, however, allow easy prediction of C_t . The study results have therefore been combined in a single overall graph in Figure 28, to which has been added the prediction line given in the Rock Manual.

That prediction line appears however excessively pessimistic for these test results, so a revised set of prediction lines have been fitted to the data from this exploratory study, plotted in Figure 28:

$$C_t = 0.8 \quad \text{for } -4 < R_c/H_s < -1.6$$

$$C_t = 0.32 - 0.3 R_c/H_s \quad \text{for } -1.6 < R_c/H_s < -0.7$$

$$C_t = 0.1 \quad \text{for } 0.7 < R_c/H_s < 3.0$$



8. FURTHER WORK

These simple exploratory tests have been more successful than anticipated in reproducing many of the failure processes, even if some tests were influenced by excessive wall restraints that may have delayed the onset of wall failure. Whilst most of the model 'arching' effects were reduced in Series 5 and 6, it is however probable that the results may have been (in part) influenced by the (large) size of blocks used. Further tests with smaller blocks might explore this effect. It would also improve confidence if any future tests (with smaller wall blocks) were to explore the influence of tidal variations on profile recession, and hence related transmission. Even so, it seems likely that the lower level of transmission presented in Figure 28 above relative to the Rock Manual prediction is realistic.

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Keywords

Breakwaters, hydraulic model tests, wave forces, wave transmission, wave shelter,

Notation

C_r	Coefficient of wave reflection, reflected wave height / incident wave height
C_t	Coefficient of wave transmission, transmitted wave height / incident wave height
D	Particle size, diameter
H_{si}	Incident significant wave height
MSL	Mean Sea Level
R_c	Crest freeboard, relative to water level
R^*	Dimensionless freeboard, $(R_c/H_{si})/(\sqrt{s(2\pi)})$
s	Wave steepness, $2\pi H_s/gT_p^2$
T_p	Peak wave period
φ	Internal friction angle
δ	Wall friction angle
ρ	Material or water density
ρ_c	Concrete density